FDM 11-10-1 Basic Criteria

December 21, 2012

1.1 Design Year

The design year is normally 20 years from the date a project is proposed to be opened to traffic. A shorter design period may be used when a highway is to be constructed in stages. A practical design period for traffic volumes for 3R projects may be five to ten years.

1.2 Traffic

Cooperate with the region's planning staff to develop design traffic data. Traffic data include current and design year average daily traffic, design hourly volumes, directional distribution, and the percentages of heavy vehicles expected in the design year. Normally trucks and buses are the heavy vehicles considered as influencing highway capacity. Also, consider heavy recreational vehicles on certain routes. Include bicycle and pedestrian counts when requesting intersection traffic counts.

1.3 Highway Capacity

Capacity is one of the most important factors in highway design and operation. Through capacity analysis, proposed highways can be designed to operate at predicted traffic volumes without exceeding a pre-selected level of service. Highways and streets are desirably designed to carry design hour traffic volumes at a Level of Service C, or better. Refer to FDM 11-5-3 for highway capacity procedures.

1.4 Functional Classification

Functional classification is the process by which streets and highways are grouped into classes or systems according to the character of service they are intended to provide. The basic functional systems used in highway planning are arterials, collectors, and locals. Using national classification terminology, these systems are sub-classified based on the trips served, the areas served, and the operational characteristics of the streets or highways. These systems are detailed on Wisconsin's current Functional Classification Systems Maps, http://dotnet/dtim-bop/function/functional.htm.

1.5 Design Speed

According to AASHTO[1], "Design speed is a selected speed used to determine the various geometric design features of the roadway. The assumed design speed should be a logical one with respect to the topography, anticipated operating speed, the adjacent land use, and the functional classification of the highway."

Selection of design speed is extremely important because this choice sets limits for curvature, sight distance, clear zone, and other geometric and cross-sectional features. The type and functional classification of highway, the topography, the adjacent land use, driver expectations, and economics are all factors influencing this selection.

Speed measurements on highways of different design speeds and various traffic volumes typically show a wide range of actual vehicle running speeds. In order to satisfy the desired travel speeds of most drivers, choose a design speed that is a high-percentile value in the speed distribution range. Average running speeds will then normally be lower than the design speed, because of the influence of traffic volumes, physical limitations of the highway, and speed limits.

Speed measurements on rural arterial highways in Wisconsin show the average running speed to be in excess of the posted speed. These studies support standard design speeds for rural arterials on the state trunk highway system that are 5 mph greater than posted speed. Table 1.1 provides corresponding english and metric design speeds with typical posted speeds.

[1] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004., p.67

Table 1.1 Design Speed vs. Typical Posted Speed

Design Speed (km/h)	Design Speed (mph)	Typical Posted Speed (mph)
50	30	25
60	35 & 40	30 or 35
70	45	40
80	50	45
90	55	50
100	60	55
110	65 & 70	60 or 65

Use a design speed of 70 mph for Corridor 2020 Backbone highways and other state trunk highways that are candidates for a posted speed limit of 65 mph. Currently the statutes limit this to multilane divided highways. Existing two-lane roadways being expanded to four lanes would be candidates for this design speed where the following conditions exist:

1. The cross section of the roadway will be divided, i.e. opposing traffic will be separated by a median or traffic barrier.
2. The planned highway meets the requirements for a freeway or expressway as defined in Section 346.57 (1) of the Wisconsin Statutes.
3. Check with the regional traffic unit to confirm that the highway segment is long enough to allow the practical use of the higher speed.
4. There are no signals or stop conditions on the highway segment.
5. The median width on an expressway equals or exceeds the clear zone width required for a 70 mph design speed. Note: It is generally not practical to use median barriers on non-access controlled highways because the barrier can obstruct driver vision at intersections and safety treating the ends of interrupted barrier can be difficult. However, the use of a median barrier would be appropriate for freeways with narrow medians.

A 70 mph (110 km/h) design speed is also used under the following conditions:

1. A posted speed of 65 mph exists on a multilane divided highway, which will be improved or extended.
2. The highway qualifies for an expansion project (2-lane to 4-lane divided) and has an accident rate that is less than 25 percent above the statewide average rate for similar types of highways, and has an 85th percentile speed of at least 60 mph.

Lower design speeds may be considered on “Special” corridors that serve more of an access, tourist or aesthetic related function than a mobility function. These “Special” corridors might be “Rustic” Roads, “Scenic Byways,” sections of urban corridors with high pedestrian activity or in the vicinity of schools, or other roadways located in unique environmentally or socially sensitive areas. Using lower design speeds can help to provide additional flexibility in the design of horizontal, vertical and cross sectional elements.

Identify these lower design speed corridors in the CDR and Scoping Phases of the project development process. The selection of the design speed needs to be mutually agreed on between the planning, traffic and project development sections. The use of lower design speeds on existing roadways must be based on an analysis of the roadway alignment and of the measured operating speeds.

On redesigned or newly designed roadways, design alignments with purposeful, curvilinear features to promote lower operating speeds that are compatible with the chosen design speeds. Also, give special consideration to corridor consistency and functional class, when selecting an appropriate design speed. See the design criteria tables in FDM 11-15-1 and FDM 11-20-1.

Regardless of the design speeds provided in FDM 11-15-1 and FDM 11-20-1, the basis for selection of a design speed must be fully documented in the Design Study Report and Exceptions to Standards Report if an exception is required. If the selected design speed equals or exceeds the legal speed limit, a statement to that effect will suffice. Otherwise this documentation shall include a discussion of the road characteristics that relate to operating speeds plus characteristics of the abutting segments of road, and statements about any advisory or regulatory speed signing in place, and discussions about practical operating speeds on the affected and abutting segments of road; or other logic that explains the basis for the selection.

Formal requests for exception to the design speed policy are not required on spot improvement projects. In this context spot improvement is defined as a project less than 0.5 mile long (e.g., a bridge replacement). Instead, choose a design speed that is consistent with both existing conditions and the planned development of the adjacent sections of highway. If this results in a project design speed that is less than the posted or statutory speed, then mitigate this to the extent possible by appropriate advisory signing, or other means.

For further information about design speed, see the section titled "Speed" in the 2004 "Speed" in the 2004 GDHS[2].

1.6 References

1. A Policy on Geometric Design of Highways and Streets 2004, 5th edition. AASHTO, Washington, DC, 2004.

FDM 11-10-5 Geometric Elements — March 4, 2013

5.1 Sight Distance

A primary feature of highway design is the arrangement of the geometric elements so that there is adequate sight distance for safe and comfortable vehicle operation. Sight distance is considered in terms of stopping sight distance, decision sight distance, passing sight distance, and intersection sight distance.

For the purpose of driveway permitting, driveway sight distance = intersection sight distance (it is also recommended that driveway sight distance be evaluated on reconstruction projects).

A consistent quality design requires that sight distance be evaluated for the entire project as a whole, rather than looking at isolated lengths of roadway. Adjustments in alignment and profile may be necessary to produce improvements in availability, distribution, and balance of sight distance along the route.

Use desirable design criteria values as the default. See FDM 11-3-5 for guidance on design criteria.

5.1.1 Stopping Sight Distance (SSD); Decision Sight Distance (DSD)

5.1.1.1 Stopping Sight Distance (SSD)

Stopping Sight Distance (SSD)[3] is the length of roadway ahead that is visible to the driver that is sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. It is the sum of two distances:

1. Brake reaction distance - the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied
2. Braking distance - the distance needed to stop the vehicle from the instant brake applications begin. Stopping distance is calculated using the 90th percentile reaction time of 2.5 seconds and the 90th percentile deceleration rate of 11.2 ft/s^2 on wet pavement.

Prior to 2001, the AASHTO GDHS[4] provided a range of values for SSD. Since 2001, the AASHTO GDHS[5] has provided a single SSD value per given design speed. Attachment 5.1 shows the required stopping sight distance for design speeds from 25-70 mph. It is desirable to adjust SSD for downgrades and use values exceeding those shown in Attachment 5.1 (see Exh. 3-2 in the 2004 AASHTO GDHS[6]).

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[2] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004., pp.66-72
[3] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004., ch. 3, pp.109-131, "Elements of Design / Sight Distance"
[4] (*2) A Policy on Geometric Design of Highways and Streets 1990*. AASHTO, 1990. www.transportation.org, ch. III, pp.117-140, "Elements of Design / Sight Distance"
[5] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004., ch. 3, pp.109-131, "Elements of Design / Sight Distance"
(*3) A Policy on Geometric Design of Highways and Streets 2001*, (2nd printing) 4th edition. AASHTO, 2001. www.transportation.org. , ch. 3, pp.109-131, "Elements of Design / Sight Distance"
[6] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.

Stopping sight distance is used when the vehicle is traveling at design speed on a wet pavement when one clearly discernable object or obstacle is presented in the roadway. Use the same SSD for trucks and cars because recent data shows that the braking distances of trucks and passenger cars on wet pavements are nearly equal[7].

5.1.1.2 Decision Sight Distance (DSD)

Decision Sight Distance (DSD) is the distance needed for a driver to detect an unexpected or otherwise difficult-to-perceive information source or condition in a roadway environment that may be visually cluttered, recognize the condition or its potential threat, select an appropriate speed and path, and initiate and complete the maneuver safely and efficiently. There are 5 categories of avoidance maneuvers identified in AASHTO (*1*):

A: Stop on rural road

B: Stop on urban road

C: Speed/path/direction change on rural road

D: Speed/path/direction change on suburban road

E: Speed/path/direction change on urban road

Design values for DSD are shown in Attachment 5.1.

Decision sight distance applies when conditions are complex, driver expectancies are different from the situation, or visibility to traffic control or design features is impaired. Complex situations create unsafe or inefficient operations because there is more information for drivers to process. Because of this, drivers need increased perception reaction time to make the proper decision. This increased time can be especially beneficial for older drivers, because they are involved in a disproportionate number of crashes where there is a higher than average demand imposed on driving skills.

5.1.1.3 Application of Stopping Sight Distance (SSD) and Decision Sight Distance (DSD)

In computing and measuring sight distance along the roadway, the height of the driver's eye is 3.5 feet above the pavement surface, and the height of the object to be seen by the driver is, as shown below, either 6-inches or 24-inches. A 6-inch object is representative of the lowest object that can create a hazardous condition and be perceived as a hazard by a driver in time to stop before reaching it[8]. A 24-inch object is equivalent to the taillight height of a passenger car because stopping is generally in response to another vehicle or large hazard in the roadway[9].

There are 3 categories of desirable and minimum standards for sight distance along a roadway[10]:

- *Category 1* - Desirable=SSD to a 6-inch object / Minimum=SSD to a 24-inch object;
- *Category 2* - Desirable=DSD for avoidance maneuver C to a 24-inch object *and* SSD to a 6-inch object* / Minimum=SSD to a 24-inch object;
- *Category 3* - Desirable=DSD for avoidance maneuver C to a 24-inch object *and* SSD to a 6-inch object* / Minimum=SSD to a 6-inch object.

  * Both conditions are necessary because providing Decision Sight Distance to a 24-inch object does not guarantee Stopping Sight Distance to a 6-inch object at all locations. Examples include roads with “roller-coaster” type vertical alignments and roads with line-of-sight obstructions on the inside of horizontal curves.

The criteria for applying these standards are shown in Attachment 5.1. Application of a particular standard is based on the complexity of the driving conditions that could be expected at a particular location. Category 1 applies to the least complex locations and is the default requirement for locations where the other categories don’t apply. Category 3 applies to the most complex locations.

Designers may use Decision Sight Distance at other locations that are not listed in Attachment 1.1 if they judge

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7 (*4) Commercial Truck and Bus Safety Synthesis Program Synthesis 3*: *Highway / Heavy Vehicle Interaction*. Transportation Research Board, 2003. http://onlinepubs.trb.org/Onlinepubs/ctbssp/ctbssp_syn_3.pdf., p.22

8 (*2) A Policy on Geometric Design of Highways and Streets 1990*. AASHTO, 1990. www.transportation.org., pp.136-137

9 (*5) NCHRP Report 400*: *Determination of Stopping Sight Distance*. TRB, National Research Council, 1997., pp. 44-45; 74-76

10 *(6) SSD and DSD Guidance. (in-house report).* Wisconsin DOT, 2006. *(7) FDM 11-10-05_20060000_SSD-DSD-computations.xls.* Wisconsin DOT, 2009.

it to be necessary. This needs to be discussed in the Design Study Report. Some examples of locations where Decision Sight Distance may be appropriate are:

- Complex operations or design features exist, including abrupt or unusual alignment changes;
- Detour Approach;
- High-speed high-volume urban arterial with considerable roadside friction.

Sight distance requirements need to be evaluated for both directions of travel on a roadway because the sight distance category might not be the same for both directions of travel.

These standards represent the requirement for all roads. It is encouraged to provide sight distance that equals or exceeds desirable standards, particularly on two-lane bi-directional roads where passing sight distance is provided if it is economically obtainable. Providing less than desirable standards but greater than or equal to minimum standards requires explanation and approval in the Design Study Report (DSR). Providing less than minimum standards requires an approved Exception to Standards per FDM 11-1-2 or FDM 11-1-4.

5.1.1.4 Sight Distance on a Stop Sign Controlled Approach

The minimum sight distance requirement along a roadway approaching a stop sign is stopping sight distance (SSD) based on the design speed of the roadway to either a 24-inch or 6-inch object, depending on the sight distance category (see FDM 11-10-5.1.1.3 and Attachments 5.1 and 5.2). The desirable horizontal and vertical sight distance requirement is as described for the sight distance category.

Another consideration, in addition to stopping sight distance, is stop sign visibility. Road users need to perceive the stop sign for a sufficient distance to respond to it. Drivers approaching a stop sign typically decelerate over a greater distance than SSD[11]. This allows a more gradual deceleration than is used for SSD. To achieve this, make sure that the stop sign is perceptible from at least the desirable upstream functional length of intersection (see FDM 11-25-1). Although it is preferable that the stop sign be visible, it may be necessary to provide other traffic control devices, such as “Stop Ahead” signs, if it is not.

Also, see FDM 11-10-5.2.2 "Horizontal Curve on a Stop Sign Controlled Approach”.

5.1.2 Sight Distance for Undercrossing

While not a frequent problem, the structure fascia may cut the line of sight on a road passing under a bridge and limit the sight distance to less than otherwise is attainable. It is generally practical to provide the minimum length of sag vertical curve at grade separation structures. Even where the recommended grades are exceeded, the sight distance must not be reduced below the minimum recommended values for sight distance. See pages 277-279 of GDHS 2004[12] for more information.

5.1.3 Passing Sight Distance

Passing sight distance is the minimum sight distance that must be available to enable the driver of one vehicle to pass another vehicle safely and comfortably, without interfering with the speed of an oncoming vehicle traveling at the design speed should it come into view after the overtaking maneuver is started. The sight distance available for passing at any location is the longest distance at which a driver, whose eyes are 3.5 feet above the pavement surface, can see an object 3.5 feet high on the road (see Attachment 5.8). See GDHS 2004[13], pages 118-126 and 270 for additional information.

The minimum passing sight distance is sufficient for single or isolated passing only, and often opposing vehicles will cancel the passing opportunity. It is desirable to provide for adequate passing sight distance over as much of the highway length as feasible. The greater the volume of traffic on the roadway the more important it is to maximize well distributed passing opportunities.

When reconstructing an existing facility it is important to achieve passing opportunity of 60 percent or greater, if possible. It may be more desirable to flatten a number of small vertical curves rather than flattening one large vertical curve.

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[11] See Wang et al in Transportation Research Record 1937 ~~(28)~~ (*8)* Normal Deceleration Behavior of Passenger Vehicles at Stop Sign-Controlled Intersections Evaluated with In-Vehicle Global Positioning System Data. In *Transportation Research Record: Journal of the Transportation Research Board*, No. *1937:* Transportation Research Board of the National Academies, 2005, pp.120-127. http://trb.metapress.com/content/u787613013238870/fulltext.pdf.;
see also AASHTO GDHS 2004 (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004., Exh. 2-25, p. 45)

[12] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.

[13] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.

Guidance on establishing, marking and signing no-passing zones can be found in the Wisconsin Traffic Guidelines Manual (TGM)[14] and is specified in the Wisconsin Standard Specifications for Highway and Structure Construction. The sight distance values shown are not to be used directly in design, but should be reviewed so that locations requiring no-passing zone markings can be recognized during design. Review proposed alignments with the region traffic section staff to assure that the passing sight distance provided will not require no-passing markings.

Passing sight design distances (AASHTO criteria) and no-passing zone standards (TGM criteria) are based on different formulas and serve different purposes. The FHWA has determined that there is no significant need to make the two distances agree. The current no-passing zone marking practice provides a balance between passing opportunities and motorist violations.

Any highway (including bypasses and expressways) that is designed as a future 4-lane divided highway, but designed to open initially as a 2-lane highway, shall be designed considering passing sight distance criteria for a 2-lane highway. The design must provide adequate passing sight distance as a 2-lane highway for its full performance life[15].

5.1.4 Intersection Sight Distance (ISD), Vision Triangles, and Vision Corners

Intersection Sight Distance is the distance for which there must be unobstructed sight along both roads of an intersection, and across their included corners, that is sufficient to allow the operators of vehicles approaching the intersection or stopped at the intersection, to safely carry out whatever maneuvers may be required to negotiate the intersection. Intersection Sight Distance is required for all at-grade intersections on all projects for both passenger cars and for the design vehicle shown in Table 5.1. Intersection sight distance is ensured by establishing a clear sight window (see Figure 5.1) across each of the included corners of an intersection. Design guidance on intersection sight distance for non-roundabout intersections can be found later in this section and is based on pages 650-677 of the 2004 AASHTO GDHS[16], but modified as noted. Guidance on intersection sight distance for roundabouts can be found in FDM 11-26-30. Guidance for intersection sight distance on 3R projects can be found in FDM 11-40-1. See FDM 11-46-20 for guidance on sight distance for trail crossings.

Table 5.1 Design Vehicle for Intersection Sight Distance $_A$

Type of Intersecting Highway	**Design Vehicle for Purposes of ISD** $_B$
Interchange ramp terminals	Combination Truck (WB-vehicle, e.g. WB-50, WB-65)
Arterials	Combination Truck (WB-vehicle, e.g. WB-50, WB-65)
Collectors	Single Unit Truck (SU-vehicle) $_C$
Local Roads / Residential Streets	Single Unit Truck (SU-vehicle) $_C$

A See FDM 11-25-2.1 for guidance on Intersection Design Vehicles and Intersection. Check Vehicles for turning movements at intersections

B Only the Passenger vehicle need be considered in areas where truck traffic is minimal (<2.5% of AADT), and right-of-way restrictions prohibit adequate sight window clearing

C If there is significant Combination Truck traffic then use that as the design vehicle instead of the Single Unit Truck.

A vision triangle is an additional clear sight window, for intersections with stop sign control on the side road and for signal controlled intersections. Its purpose is to provide an opportunity for speed adjustment or evasive maneuver by a vehicle on the major highway if a vehicle on the minor road violates the traffic control. In other words, a Vision Triangle is a supplement to, and not a substitute for intersection sight distance. ISD must be provided at all intersections whether or not a vision triangle is provided. Guidance on vision triangles and where to use them can be found later in this section. Guide dimensions for vision triangles can be found in Attachment 5.13.

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[14] (*9)* Applications / No Passing Zone Standards. In *WisDOT Traffic Guidelines Manual (TGM) ch. 3: Markings* 2009, sect. 3-2-2, pp.1-5. http://dotnet/dtid_bho/extranet/manuals/tgm/03/03-02-02.pdf.
[15] *(10) Review of Wisconsin Bypass Road Design Practices February 13-14, 2006. (Final).* Federal Highway Administration Resource Center, 2006. http://www.dot.wisconsin.gov/library/publications/docs/wis-bypass-report.pdf.
[16] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.

A "vision corner" is defined as either

- The clear sight window for intersection sight distance, if no vision triangle is used, or
- The combination of the clear sight window for ISD and the clear sight window for vision triangle, as shown in Figure 5.1.

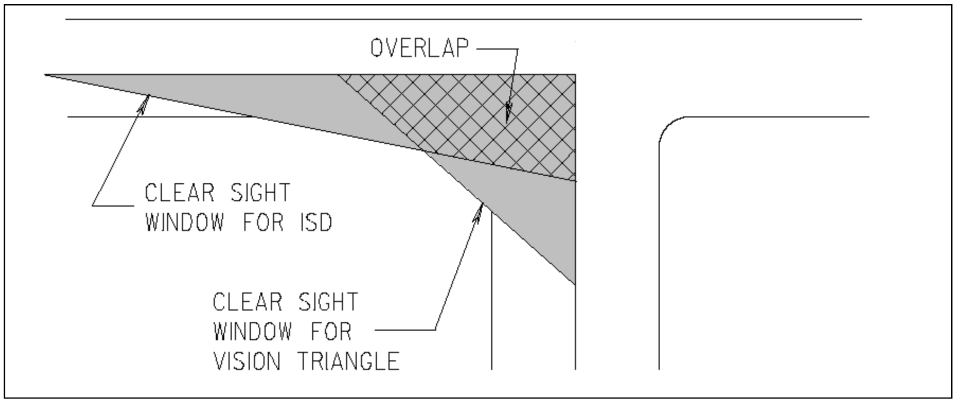


***Figure 5.1* Example of Vision Corner**

5.1.4.1 Clear Sight Window

A clear sight window and its horizontal and vertical boundaries are shown in Figure 5.2. The dimensions of a clear sight window vary depending on the type of vehicle and the type of intersection control (see Table 5.1 above and Design Guidance for Intersection Sight Distance below).

In establishing sight lines through a clear sight window, use an eye height above the roadway surface of 3.5 feet for passenger cars and 7.6 feet for trucks. Use an object height above the roadway surface of 3.5 feet.

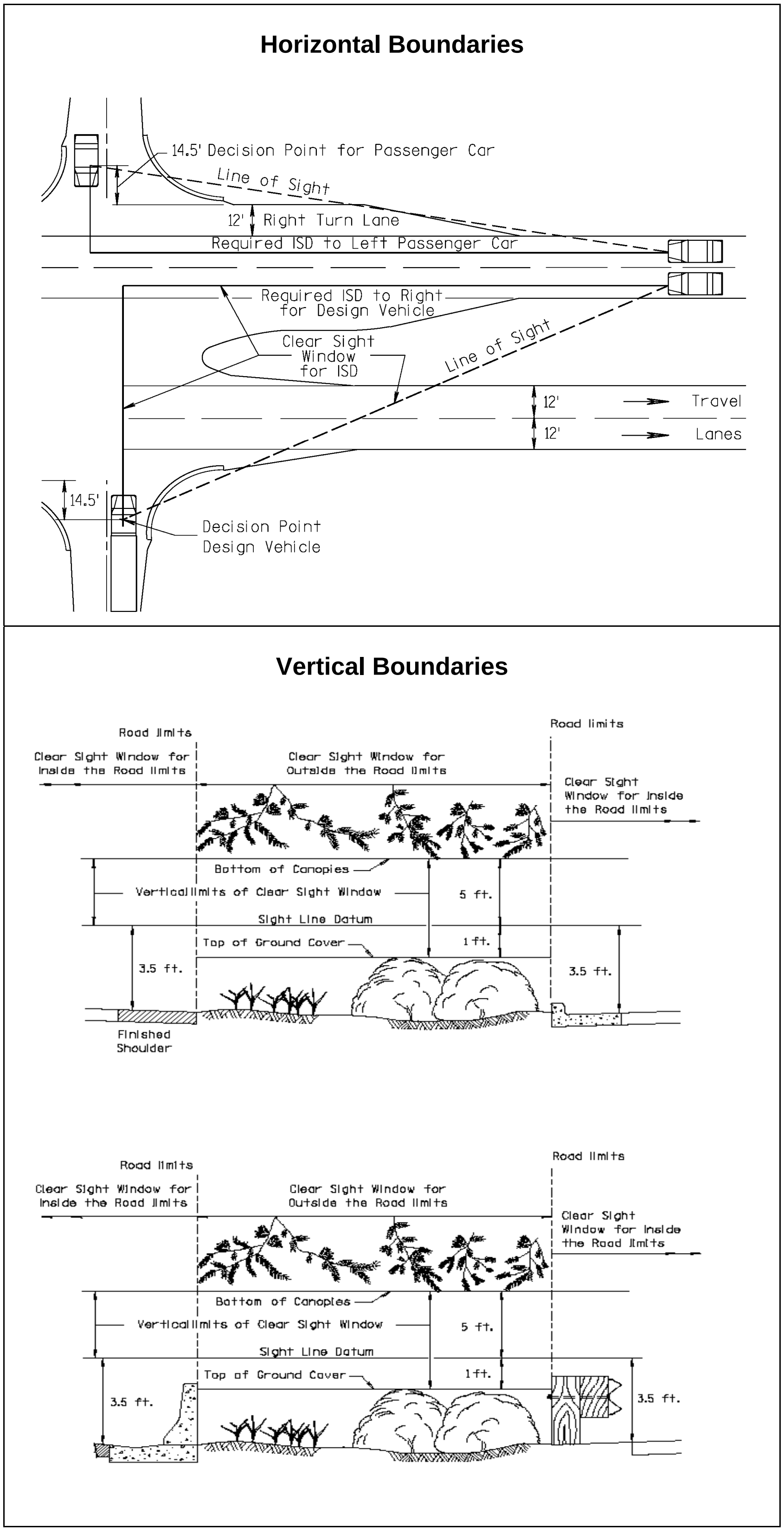


Figure 5.2 Clear Sight Window (adapted from Florida DOT[17])

5.1.4.1.1 Horizontal Boundaries of a Clear Sight Window

The horizontal boundary of a clear sight window on the mainline is the center of the approach travel lane,

17 *Adapted from (11) Sight Distance at Intersections. In FLDOT Road Design Detail Florida DOT, 1989, Index No. 546.*

beginning at the intersection and ending at a point known as the decision point for the mainline- established by applying the intersection sight distance requirements for the intersection control case (see FDM 11-10-5.1.4.2 “Design Guidance for Intersection Sight Distance” below).

The horizontal boundary of a clear sight window on the side road is the center of the approach travel lane, beginning at the intersection and ending at a point known as the decision point for the side road - established by applying the intersection sight distance requirements for the intersection control case (see FDM 11-10-5.1.4.2 “Design Guidance for Intersection Sight Distance” below).

The horizontal boundary of a clear sight window across the included corner of an intersection is the line connecting the end points, or decision points, of the first two sides.

5.1.4.1.2 Vertical Boundaries of a Clear Sight Window

The bottom boundary of a clear sight window is the sight line datum inside the road limits, and 1 foot below the sight line datum outside the road limits. A sight line datum is defined as a line of sight which is 3.5-feet above the pavement surface on each end. The road limit is defined as the edge of finished shoulder or the back of curb, whichever is applicable, unless there is a barrier or bridge parapet. In that case the road limit is defined as the back edge of roadside barrier or bridge parapet.

The top boundary of a clear sight window is 5-foot above the sight line datum.

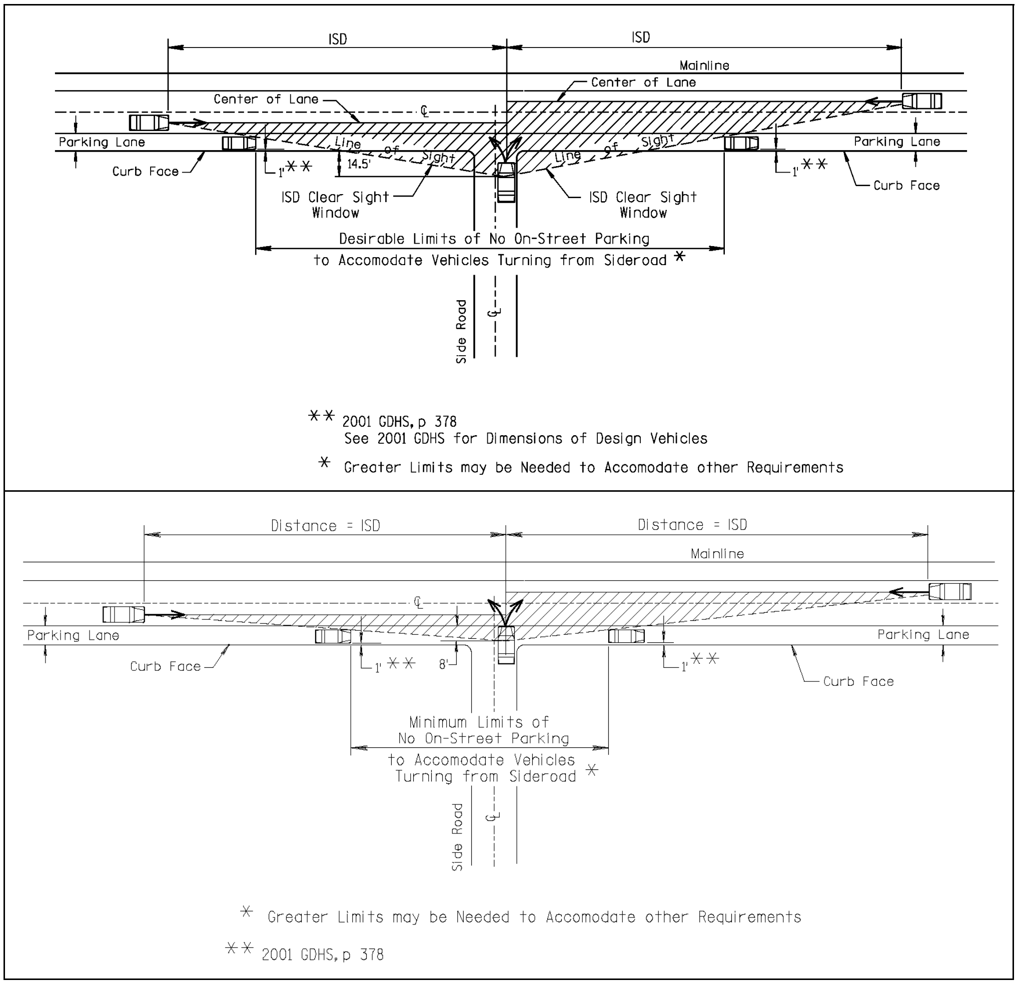
5.1.4.1.3 Obstructions Within a Clear Sight Window

Make sure that a clear sight window is clear of obstructions that might block a driver’s view of potentially conflicting vehicles. This includes, but is not limited to:

- The roadway itself - check the vertical alignment and superelevation of the highway to see if the pavement obscures the line of sight,
- Roadside and median barriers – including beamguard,
- Bridge parapets and railings,
- Cut slopes and embankments,
- On-street parked vehicles - See Figure 5.3. Prohibit on-street parking as follows, unless greater restrictions are required by statute, the Wisconsin MUTCD, or to provide adequate lines of sight for pedestrians:
  - **Desirably**: within the Intersection Sight Distance clear sight windows[18].
  - **Minimally**: so that a vehicle entering from the sideroad does not have to encroach on the mainline travel lane or bicycle lane in order for the driver to see a vehicle approaching on the mainline at a distance equal to ISD.
  - **NOTE**: assume parked vehicles are 12 inches from the curb face.
- Off-street parked vehicles - Prohibit off-street parking within the ISD clear sight windows,
- Signal control cabinets,
- Landscaping,
- Signs - offset signs to prevent sight distance obstructions,
- Structures, including, but not limited to, buildings, fences, retaining walls, screening,
- Vegetation, including bushes, hedges, natural growth, plantings, tall crops, tree branches, and tree trunks.

There must be sufficient right-of-way to ensure that line-of-sight obstructions can be removed.

[18](12)*Access Management Manual*. Transportation Research Board, 2003., Figure 5.8-5.9



***<u>Note</u>: This is not an alternative method for establishing ISD. Parked cars are not permanent sight obstructions. ISD from 14.50 feet behind the near face of mainline curb is still required per FDM 11-10-5.1.4.2.2.

Figure 5.3 Determining On-Street Parking Limits*

Street markers, traffic signs, and other traffic controls are allowed within the Clear Sight window, provided their number and arrangement do not significantly block vision across the area. Likewise, utility pedestals and poles are allowed within the Clear Sight window, provided their number and arrangement do not significantly block vision across the area. Consider offsetting the right turn lane at intersections where there are a significant number of right turns that impede clear sight lines.

Urban Intersections can be a particular concern because of the abundance of street furniture and development in the vicinity of an intersection. Pay particular attention to the potential for line of sight obstruction in the placement of signs, light poles, signal controllers, tree plantings, newspaper and advertising boxes, etc.

Pedestrian Considerations: Features such as landscaping, parked cars, utility poles, traffic control devices, and street furniture can create sight obstructions for pedestrians. Consider installing curb extensions or instituting parking restrictions to ensure that pedestrian sight lines are not blocked.

5.1.4.2 Design Guidance for Intersection Sight Distance

As mentioned above, intersection sight distance is required for all at-grade intersections on all projects for both passenger cars and for the design vehicle shown in Table 5.1. Guidance is provided below for the following intersection control cases.

- Case A - Intersections with no control
- Case B - Intersections with stop control on the minor road
    - B1 - Left turn from the minor road
    - B2 - Right turn from the minor road
    - B3 - Crossing maneuver from the minor road

- Case C - Intersections with yield control on the minor road
  - C1 - Crossing maneuver from the minor road
  - C2 - Left or right turn from the minor road
- Case D - Intersections with traffic signal control
- Case E - Intersections with all-way stop control
- Case F - Left turns from the major road

5.1.4.2.1 Case A - Intersections with No Control

Do not allow an uncontrolled at-grade intersection with a STH. For non-STH locations, use guidance from GDHS 2004[19], pages 654-657.

5.1.4.2.2 Case B - Intersections with Stop Control on the Minor Road

Gap Acceptance, as described on page 659, GDHS 2004[20] and modified herein, is the basis for computing Case B Intersection Sight Distances.

Decision point location for the side road vehicle is the position of the side road driver's eye in relation to the mainline. For rural intersections, this distance approximates the location of a passenger car at a standard stop bar installation. It is equal to -14.50 feet from the farthest outside edge of non-shoulder mainline pavement. In most situations, this would be either the edge of the mainline right turn lane, the mainline right-turn taper, the mainline downstream acceleration taper, or the mainline travel lane. However, if there is a separate channelized right-turn lane on the mainline, this distance would be to either the mainline downstream acceleration taper, or the edge of the near mainline travel lane.

For urban intersections, this distance is 14.50 feet-from the near face of mainline curb.

Decision Point Location for crossing vehicles stopped in a median is the position of the driver's eye in relation to the far side traffic lanes. For medians without a stop bar, assume the vehicle stops with its front end 3 feet from the median edge of travel lane. The driver's eye is 8 feet behind this, or 11 feet from the median edge of travel lane. For medians with a stop bar this distance = 8.0 feet behind the stop bar.

Median width needs to be at least 6 feet greater than vehicle length for a vehicle to complete a crossing in two (2) steps.[21].

Decision point location for the mainline vehicle varies by design speed and design vehicle. Table 5.2 shows the Intersection Sight Distance requirements for AASHTO Intersection Control Cases B1, B2, and B3. See Attachment 5.14 for an example computation.

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[19] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.
[20] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.
[21] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004, pp 456-457; (*13) NCHRP Report 375*: *Median Intersection Design*. TRB, National Research Council, 1995. pp 67-68; (*14)* Sight Distance / Stop Control on Cross Street. In *NJDOT Roadway Design Manual-2008 ch. 6: At-Grade Intersections* New Jersey DOT, 2008, section 6.3.3. http://www.nj.gov/transportation/eng/documents/RDM/sec6.shtm#stopcross.

Table 5.2 Intersection Sight Distance [A] Requirements for Intersection Control Cases B1, B2, and B3 - Stop on Minor Road[22]

	Case B1 - Left turn from the minor road [B]			Case B2 - Right turn from the minor road [C]			Case B3 - Crossing maneuver from the minor road [D]		
Design vehicle	**P**	**SU**	**WB**	**P**	**SU**	**WB**	**P**	**SU**	**WB**
Eye height (ft)	3.5	7.6	7.6	3.5	7.6	7.6	3.5	7.6	7.6
Time gap (sec) **DESIRABLE** (MINIMUM)	**10.0** (7.5)	**12.0** (9.5)	**13.0** (11.5)	**8.0** (6.5)	**10.0** (8.5)	**12.0** (10.5)	**7.0** (6.5)	**10.0** (8.5)	**13.0** (10.5)
Mainline Design Speed (mph)	**ISD (ft) DES** (MIN)	**ISD (ft) DES** (MIN)	**ISD (ft) DES** (MIN)	**ISD (ft) DES** (MIN)	**ISD (ft) DES** (MIN)	**ISD (ft) DES** (MIN)	**ISD (ft) DES** (MIN)	**ISD (ft) DES** (MIN)	**ISD (ft) DES** (MIN)
25	**370** (280)	**445** (350)	**480** (425)	**295** (240)	**370** (315)	**445** (390)	**260** (240)	**370** (315)	**480** (390)
30	**445** (335)	**530** (420)	**575** (510)	**355** (290)	**445** (375)	**530** (465)	**310** (290)	**445** (375)	**575** (465)
35	**515** (390)	**620** (490)	**670** (595)	**415** (335)	**515** (440)	**620** (545)	**365** (335)	**515** (440)	**670** (545)
40	**590** (445)	**710** (560)	**765** (680)	**475** (385)	**590** (500)	**710** (620)	**415** (385)	**590** (500)	**765** (620)
45	**665** (500)	**795** (630)	**860** (765)	**530** (430)	**665** (565)	**795** (695)	**465** (430)	**665** (565)	**860** (695)
50	**735** (555)	**885** (700)	**960** (850)	**590** (480)	**735** (625)	**885** (775)	**515** (480)	**735** (625)	**960** (775)
55	**810** (610)	**975** (770)	**1055** (930)	**650** (530)	**810** (690)	**975** (850)	**570** (530)	**810** (690)	**1055** (850)
60	**885** (665)	**1060** (840)	**1150** (1015)	**710** (575)	**885** (750)	**1060** (930)	**620** (575)	**885** (750)	**1150** (930)
65	**960** (720)	**1150** (910)	**1245** (1100)	**765** (625)	**960** (815)	**1150** (1005)	**670** (625)	**960** (815)	**1245** (1005)
70	**1030** (775)	**1235** (980)	**1340** (1185)	**825** (670)	**1030** (875)	**1235** (1085)	**725** (670)	**1030** (875)	**1340** (1085)

[22] *(15) NCHRP Report 383*: *Intersection Sight Distance*. TRB, National Research Council, 1996.
(16) Intersections at Grade. In *FHWA-RD-01-051: Guidelines and Recommendations to Accommodate Older Drivers and Pedestrians* Federal Highway Administration Turner-Fairbank Research Center, 2001, Section I. http://www.tfhrc.gov/humanfac/01105/cover.htm;http://www.tfhrc.gov/humanfac/01105/01-051.pdf.
(17) ISD and vision triangle recomm vs 1990 GDHS.xls. Wisconsin DOT, 2004.

[A] Intersection Sight Distance = time gap x design speed in ft/s. ft/s = mph x (5280 ft per mi divided by 3600 sec per hour). Table values have been rounded.

[B] Case B1 Time gaps and intersection sight distances are for a stopped vehicle to turn left onto a two-lane highway with no median and grades 3 percent or less. The table values require adjustment as follows:

- For multilane highways. For left turns onto two-way highways with more than two lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, from the left in excess of one, to be crossed by the turning vehicles. Medians are computed as equivalent lane widths if they are too narrow for a vehicle to stop in, e.g. a 30-ft median would be equivalent to 2.5 lanes. Mainline right turn lanes and tapers are also treated as equivalent lane widths.
- For right turns, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, from the left in excess of zero, to be crossed by the turning vehicles. Mainline right turn lanes and tapers are also treated as equivalent lane widths.
- For minor road approach grades. If the approach grade is an upgrade that exceeds 3 percent then add 0.2 seconds for each percent grade for left turns.
- For skew. Use guidance from page 677, GDHS 2004[23].

[C] Case B2 Time gaps and intersection sight distances are for a stopped vehicle to turn right onto a two-lane highway with grades 3 percent or less. The table values require adjustment as follows:

- Add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, from the left in excess of zero, to be crossed by the turning vehicles. Mainline right turn lanes and tapers are treated as equivalent lane widths.
- For minor road approach grades. If the approach grade is an upgrade that exceeds 3 percent then add 0.1 seconds for each percent grade.
- For skew. Use guidance from page 677, GDHS 2004

[D] Case B3 Time gaps and intersection sight distances are for a stopped vehicle to cross a two-lane highway with no median and grades 3 percent or less. The table values require adjustment as follows:

- For multilane highways. For crossing a major road with more than two lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane to be crossed and for narrow medians that cannot store the design vehicle. Medians are computed as equivalent lane widths if they are too narrow for a vehicle to stop in, e.g. a 30-ft median would be equivalent to 2.5 lanes. Mainline right turn lanes and tapers are also treated as equivalent lane widths.
- For minor road approach grades. If the approach grade is an upgrade that exceeds 3 percent then add 0.1 seconds for each percent grade.
- For skew. Use guidance from page 677, GDHS 2004

5.1.4.2.3 Case C - Intersections with Yield Control on the Minor Road

Except for roundabouts, do not allow a yield-controlled at-grade intersection on a STH. Use the guidance from pages 666-673, GDHS 2004 for non-roundabout yield-controlled intersections on non-STH roads - except use Case B1 and B2 values, per Table 5.2, for mainline ISD distances for Case C2 - Left or right turn from the minor road.

Refer to the WisDOT Roundabout Guide for guidance on Intersection Sight Distance for roundabouts.

5.1.4.2.4 Case D - Intersections with Traffic Signal Control

Use the guidance from pages 671, 673, GDHS 2004, except use the values from Table 5.2 where Case B is called for, e.g. where right turn on red is allowed.

5.1.4.2.5 Case E - Intersections with All-Way Stop Control (AWSC)

At intersections with all-way stop control, the first stopped vehicle on one approach should be visible to the drivers of the first stopped vehicles on each of the other approaches. There are no other sight distance criteria applicable to intersections with all-way stop control. See page 674, GDHS 2004[24].

5.1.4.2.6 Case F - Left Turn from the Major Road

Use the guidance from pages 674-676, GDHS 2004, except use the time gaps and Intersection Sight Distance shown in Table 5.3.

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[23] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.
[24] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.

Provide a positive offset for opposing left turn lanes, if possible. A positive left-turn lane offset, as shown in Figure 5.4, can be helpful in allowing drivers in opposing left-turn bays to see past each other to detect oncoming traffic. It can also improve the operations of signalized intersections by allowing more efficient use of permissive left-turn phasing. See FDM 11-25-5, “Slotted Left-turn Lanes” for additional guidance.

Table 5.3 Intersection Sight Distance (ISD) Requirements for Case F - Left Turn from Major Road[25]

Design vehicle	P	SU	WB
Eye height (ft)	3.5	7.6	7.6
Time gap [A] (sec) **DESIRABLE** (MINIMUM)	**8.0** (5.5)	**8.0** (6.5)	**8.0** (7.5)
Mainline Design Speed (mph)	**ISD (ft)** **DES** (MIN)	**ISD (ft)** **DES** (MIN)	**ISD (ft)** **DES** (MIN)
25	295 (205)	**295** (240)	**295** (280)
30	**355** (245)	**355** (290)	**355** (335)
35	**415** (285)	**415** (335)	**415** (390)
40	**475** (325)	**475** (385)	**475** (445)
45	**530** (365)	**530** (430)	**530** (500)
50	**590** (405)	**590** (480)	**590** (555)
55	**650** (445)	**650** (530)	**650** (610)
60	**710** (490)	**710** (575)	**710** (665)
65	**765** (530)	**765** (625)	**765** (720)
70	**825** (570)	**825** (670)	**825** (775)

[A] Time gaps and intersection sight distances are for a vehicle making a turn left from an undivided 2-lane highway (1 lane in each direction). The table values require adjustment as follows: For left-turning vehicles that cross more than one opposing lane, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane to be crossed. Median width crossed is computed as equivalent lane widths, and is measured from the inside edge of the left turn lane to the median edge of the opposing travel lane(s).

[25] *(16)* Intersections at Grade. In *FHWA-RD-01-051: Guidelines and Recommendations to Accommodate Older Drivers and Pedestrians* Federal Highway Administration Turner-Fairbank Research Center, 2001, Section I. http://www.tfhrc.gov/humanfac/01105/cover.htm;http://www.tfhrc.gov/humanfac/01105/01-051.pdf.

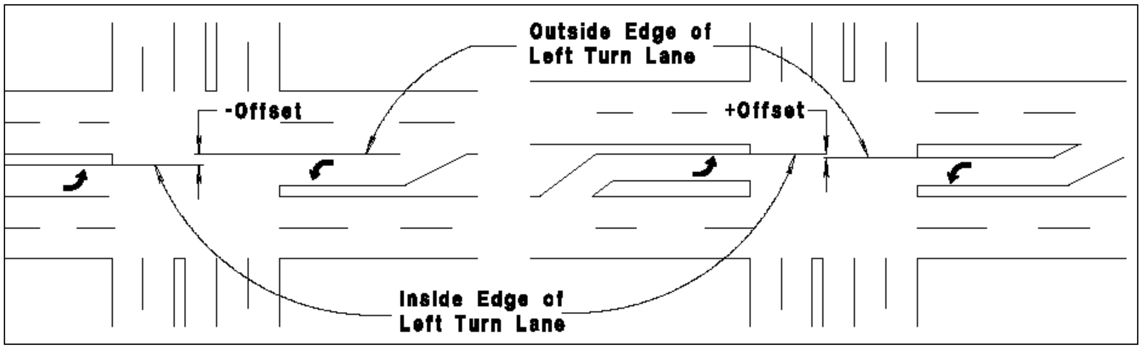

Figure 5.4 Negative (-) Offset and Positive (+) Offset of Opposing Left Turn Lanes[26]

5.1.4.2.7 Interchange Ramp Terminals at Crossroads

Treat ramp terminals at crossroads like any other at-grade intersection. Provide intersection sight distance based on the applicable intersection control case as described above. However, be very conscious of potential sight obstructions such as bridge railings, piers, and abutments that are likely to be found near ramp terminals. Design the crossroad profile - and also provide enough separation between the ramp terminal intersection and the structure - so that intersection sight distance is not obstructed.

If the crossroad is an undercrossing, check the sight distances under the structure graphically using the appropriate eye height for the vehicle at the intersection, and an object height of at least 2.0 feet.

Traffic control such as signals, all-way stop signs, or a roundabout may be a possible solution for ramp terminal locations where there is inadequate sight distance because less intersection sight distance is needed.

5.1.4.3 Design Guidance for Vision Triangles[27]

Where there is stop sign or traffic signal control on the side road, the required clear sight window for intersection sight distance extends only a short distance along the side road because it is assumed that the driver of a vehicle approaching on the side road will see and obey the traffic control. However, there are several hundred thousand crossing path crashes every year in this country caused by drivers running stop signs and red lights[28]. Adding a vision triangle enhances the safety of an intersection by providing an opportunity for speed adjustment or evasive maneuver by a vehicle on the major highway in the event a vehicle on the minor road violates a traffic control. This safety enhancement can be particularly important at higher volume intersections on high-speed roads.

As mentioned above, a vision triangle is an additional clear sight window for intersections with stop sign control on the side road and for signal controlled intersections. In other words, a vision triangle is a supplement to, and not a substitute for, the Intersection Sight Distance (ISD). **ISD must be provided at all intersections whether or not a vision triangle is provided.**

Guide dimensions for vision triangles are shown on Attachment 5.13. The dimensions are based on providing two seconds of travel time at the posted speed+5 mph for both the mainline and the side road, i.e. they are reciprocal with respect to the time both drivers can see and react to each other. Greater dimensions may be used if desired - e.g. if local zoning ordinances show greater distances. On the other hand, if site conditions, such as a building taking or unacceptable environmental impacts, preclude obtaining the recommended triangle, a partial vision triangle can still be beneficial.

Do not rely on vision triangles as the sole protection against run-the-stop-sign crashes. Measures such as increasing visibility of traffic control devices and providing streetlights can aid approaching drivers in detecting an intersection and its controls.

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[26] *(18) Median Handbook*. Florida DOT, 2001, Ch 3, p8

[27] *(17) ISD and vision triangle recomm vs 1990 GDHS.xls.* Wisconsin DOT, 2004.

[28] References: *(19) Intersection Collision Avoidance Using ITS Countermeasures.* NHTSA, 2000. http://www.itsdocs.fhwa.dot.gov/JPODOCS/REPTS_TE/13341.pdf, p6.10; *(20) Intersection Decision Support Project: Taxonomy of Crossing-Path Crashes at Intersections Using GES 2000 Data.* University of California Traffic Safety Center, 2003. http://repositories.cdlib.org/cgi/viewcontent.cgi?article=1006&context=its/tsc., pp 4-6, 11 *(21) Reducing Crashes at Rural Thru-Stop Controlled Intersections.* CH2M Hill, 2002. http://www.ctre.iastate.edu/pubs/midcon2003/PrestonIntersections.pdf."

Provide vision triangles as shown below. If it is not possible to provide a vision triangle, justification is required in the Design Study Report (DSR). Vision triangles are not used at roundabouts because research has shown that excessive intersection sight distance at roundabouts results in a higher frequency of crashes.

5.1.4.3.1 Criteria for Providing Vision Triangles

The following criteria are for STHs for new construction and reconstruction projects, for new intersections or driveways which are not part of a project, and for proposed subdivisions which include or abut an intersection or driveway.

- DO NOT provide a vision triangle at a roundabout
- Provide at at-grade intersections on expressways.
- Provide at other intersections and at driveways which meet either of the following warrants:
  - the posted speed of the STH ≥45 mph, and current or construction year traffic volume on the STH >750 AADT, and current or construction year traffic volume on the side road (or driveway) >400 AADT, and the sum of both >1250 AADT.
  - the posted speed of the STH ≥45 mph, and design year traffic volume on the STH >2500 AADT and design year traffic volume on the side road (or driveway) >1000 AADT.
- Provide at intersections where there has been a history of run-the-stop sign or run the red-light crashes.
- Perpetuate at intersections where they have been previously provided.
- Vision triangles are optional at other intersections and driveways.

The following criteria are for vision triangles on *STH 3R projects*.

- DO NOT provide a vision triangle at a roundabout
- Provide at intersections which are being upgraded, and which meet either of the following warrants:
  - the posted speed of the STH ≥45 mph, and current or construction year traffic volume on the STH >750 AADT, and current or construction year traffic volume on the side road (or driveway) >400 AADT, and the sum of both >1250 AADT.
  - the posted speed of the STH ≥45 mph, and design year traffic volume on the STH >2500 AADT, and design year traffic volume on the side road >1000 AADT.
- Provide at intersections where there has been a history of run-the-stop sign or run-the red-light crashes.
- Perpetuate at intersections where they have been previously provided.
- Vision triangles are optional at other intersections and driveways.

5.1.4.3.2 Criteria for Providing Vision Triangles on STHs for Preventive Maintenance Projects

- DO NOT provide a vision triangle at a roundabout
- Vision triangles are optional at other intersections.

5.1.4.3.3 Land Rights and/or Interests for Vision Triangles

See FDM 12-1-15 for definitions of the various types of land rights and interests acquired by the Department. In order of preference, land rights or interests for vision triangles can be:

1. Fee Title,
2. Restricted Development Easement--when a fee title interest will have a significant adverse impact on the parcel.
3. None -This only applies to vision triangles that are established by local zoning ordinances, since these are not necessarily dedicated as road right-of-way. In this case enforcement is through the local zoning authority.

5.1.4.4 Mitigation Measures for Sight Distance Deficiencies at Intersections

Some intersections may have either inadequate intersection sight distance and/or inadequate roadway sight distance approaching the intersection that is causing safety and operational problems. Ideally, the deficiency would be corrected in a timely and cost-effective manner. However, this is not always possible. Consult with the Region Traffic Section on possible mitigation measures. Some mitigation measures that might be considered-either alone or in combination-are:

- Restrict or prohibit some turning movements.
- Reduce the regulatory speed. Use appropriate signing and warning lights.

- All-way stop sign control, traffic signals, or roundabout - these require that there be adequate sight distance on the roadways approaching the intersection.
- Provide travel lane rumble strips (typically for approaching a stop sign). Be aware of noise and proximity to houses.
- Provide advance signing, and possibly warning lights.
- Adjust signing at intersection so that it is visible from farther away. Do this by making the signs larger and/or brighter and/or providing additional signs and marking.
- Provide pork chop islands on sideroad approaches to allow for more effective placement of supplemental STOP signs or traffic signals, and to encourage better positioning of stopped vehicles for enhanced visibility of approaching mainline traffic.
- Provide street lighting at the intersection. Light poles also provide daytime recognition of the presence of intersections.
- Use “CROSS TRAFFIC DOES NOT STOP” signs at intersections that drivers could misinterpret as an all-way stop (for example, two roads of equal status intersect and only one of them has a stop sign).

See the following references for additional safety measures and mitigation[29]:

- NCHRP Report 500, vol. 5 and vol. 12,
- FHWA bypass report,
- FHWA-SA-09-020, “Low-Cost Safety Enhancements for Stop-Controlled and Signalized Intersections”

5.1.4.5 Sight Distance for Railroad-Highway Grade Crossings

See FDM Chapter 17 Railroad Coordination.

5.2 Horizontal Alignment[30]

Horizontal alignment should be as straight as possible and consistent with environmental, physical, and economic constraints. Whenever feasible, avoid using maximum curvature. Flatter curvature with shorter tangents is generally preferable to sharp curves connected by long tangents. Alignment must be consistent. Sudden changes from flat to sharp curves and long tangents followed by sharp curves create safety hazards. Likewise, avoid using reverse curves unless a sufficient length of tangent is included between the curves to provide for superelevation transition. Also, avoid using very long curves because they inhibit some drivers from making passing maneuvers even when adequate sight distance exists.

The horizontal alignment development process can possibly introduce trial alignments that have curvature, superelevation, or superelevation transition carried on to or through a structure. Such alignments should be avoided, except when there is a definite need, or a specific purpose. These situations almost always result in an unsightly appearance of the bridge or bridge railing, and create needless complications in design and construction. Safety considerations are paramount however, and shall not be sacrificed to meet the foregoing criteria. If the designer feels a bridge must be built on superelevation or transition, the reasons for this should be explained in the Structure Survey Report.

A horizontal curve should not be introduced near the crest of a vertical curve. The combination of horizontal and vertical curves can greatly reduce sight distance creating hazardous conditions. These conditions hide the horizontal curve from the approaching driver, especially at night. The hazard can be avoided by having the horizontal curvature lead the vertical curvature; i.e., the horizontal curve is made longer than the vertical curve. Although the designer must attempt to optimize the horizontal alignment with respect to other factors and avoid the appearance of inconsistencies and distortion in the alignment, the horizontal alignment should be

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[29] (*22) NCHRP Report 500*: *A Guide for Addressing Unsignalized Intersection Collisions*, vol 5 of Guidance for Implementation of the AASHTO Strategic Highway Safety Plan. Transportation Research Board of the National Academies, 2003. http://onlinepubs.trb.org/Onlinepubs/nchrp/nchrp_rpt_500v5.pdf.
(*23) NCHRP Report 500*: *A Guide for Reducing Collisions at Signalized Intersections*, vol 12 of Guidance for Implementation of the AASHTO Strategic Highway Safety Plan. Transportation Research Board of the National Academies, 2004. http://onlinepubs.trb.org/Onlinepubs/nchrp/nchrp_rpt_500v12.pdf.
(10) Review of Wisconsin Bypass Road Design Practices February 13-14, 2006. (Final). Federal Highway Administration Resource Center, 2006. http://www.dot.wisconsin.gov/library/publications/docs/wis-bypass-report.pdf.
(24) Low-Cost Safety Enhancements for Stop-Controlled and Signalized Intersections. FHWA-SA-09-020. Federal Highway Administration, 2009.
http://safety.fhwa.dot.gov/intersection/resources/fhwasa09020/fhwasa09020.pdf.

[30] ((*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004., ch. 3, pp.131-231, “Elements of Design / Horizontal Alignment”

coordinated with the vertical and cross-sectional features of the highway.

Adequate sight distance must also be provided on horizontal curves. If an object off the pavement such as a bridge pier, cut slope, or natural growth restricts sight distance then the minimum radius of curvature is determined considering the required sight distance and the lateral clearance to the object. Attachment 5.9 shows the relationships between the obstruction, degree of curve, design speed, and sight distance. If horizontal sight distance is not achieved then an Exception to Standards is required in accordance with FDM 11-1-2.

Although the use of P.I.'s without accompanying horizontal curves is discouraged, there may be situations where it is necessary. This must be discussed and justified in the DSR. Table 5.4 shows maximum deflection without a horizontal curve.

Table 5.5 shows the maximum deflection through a low-speed urban intersection – both for lane shifts and for centerline deflections. Short curves may be desirable at each end of a lane shift, especially if pavement markings are used through the intersection to provide positive guidance to the motorist.

If possible, avoid a lane shift at a signalized intersection - particularly where mounting signal heads over each lane. It complicates the design and may confuse drivers. Also, avoid deflections thru signalized intersections if possible. Also, avoid where lane designation signs are used.

Table 5.4 Maximum Deflection Without Horizontal Curves [31]

Posted Speed (S) mph		Deflection Δ* Desirable	Deflection Δ* Maximum
Low Speed	25	3° 45'	5° 30'
	30	2° 45'	3° 45'
	35	2° 15'	2° 45'
	40	1° 45'	2° 15'
High Speed	45	1° 15'	1° 15'
	50	1° 00'	1° 15'
	55	1° 00'	1° 00'
	60	0° 45'	1° 00'
	65	0° 45'	0° 45'

* Rounded to nearest 15'

Use desirable, except that maximum may be used to MATCH an existing condition.

Based on the following formulas:

Desirable

- Low Speed: $\tan \Delta = 60 / (S+5)^2$
- High Speed: $\tan \Delta = 1.0 / (S+5)$

Maximum

- Low Speed: $\tan \Delta = 60 / S^2$

[31] Adapted from (*25)* Horizontal Alignment - Maximum Centerline Deflection without Horizontal Curve. In *OHDOT Location & Design Manual, Vol.1, Roadway Design ch. 200: Horizontal and Vertical Design* Ohio DOT, 2006, sect. 202.2, pp.5, Fig. 202-1E. http://www.dot.state.oh.us/Divisions/ProdMgt/Roadway/roadwaystandards/Location%20and%20Design%20Manual/200_jul08.pdf.; and from
(*26)* Temporary Traffic Control Elements - Tapers. In *Manual on Uniform Traffic Control Devices ch. 6: Temporary Traffic Control* Federal Highway Administration, 2003, sect. 6C.08, pp.6C-5-6C-8. http://mutcd.fhwa.dot.gov/pdfs/2003/pdf-index.htm.

- High Speed: TAN $\Delta = 1.0 / S$

Where:

S = Posted Speed

Δ = Deflection Angle

Minimum distances between consecutive horizontal deflections (i.e., P.I.'s) is:

- Low Speed: 100'
- High Speed: 200'

Table 5.5 Maximum Deflection for Through Lanes Through Urban Intersections

Posted Speed	**25**	**30**	**35**	**40**
Maximum Deflection	7^0 30'	5^0 30'	4^0 15'	3^0 15'

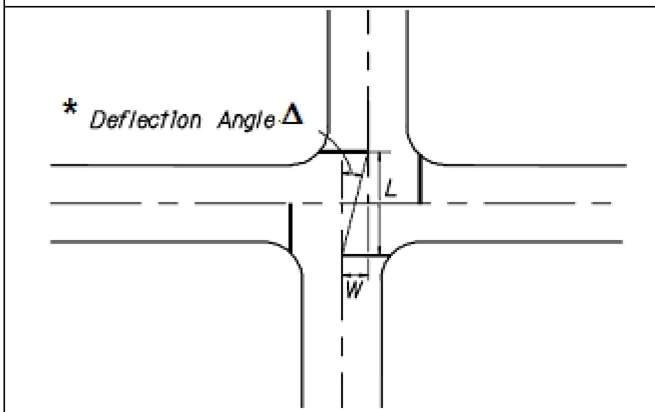
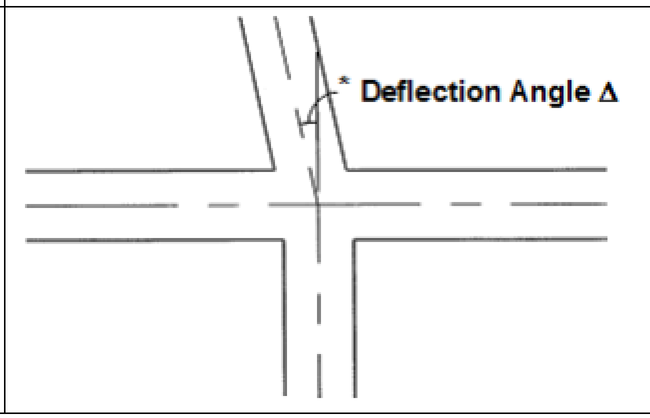
Lane Shift	**Deflection Thru Intersection**
	
* TAN $\Delta = 120 /(S+5)^2$	

Figure 5.5 Deflection Angle [32]

5.2.1 Reference Line

The basic highway reference line should be the centerline of a normal two-way roadway. The basic reference line of divided highways may be located either along the centerline of the median or along the median edge of the right-hand through pavement in the direction of stationing. All stationing and profiles of finished grade and original ground should be referred to the basic highway reference line. An auxiliary reference line along the median edge of the left-hand pavement may be desirable when roadways are not parallel or concentric, or are widely separated.

Stationing of projects (main line and side roads) should be from west to east or south to north based on the cardinal direction of the overall highway route, not just the portion of the highway within the project under design.

5.2.2 Horizontal Curve on a Stop Sign Controlled Approach

A horizontal curve in close proximity to the intersection on a stop-sign controlled approach, as shown in Figure 5.6, needs to accommodate a reasonable operating speed, while minimizing the potential for adverse operations

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[32] *(26)* Temporary Traffic Control Elements - Tapers. In *Manual on Uniform Traffic Control Devices ch. 6: Temporary Traffic Control* Federal Highway Administration, 2003, sect. 6C.08, pp.6C-5-6C-8. http://mutcd.fhwa.dot.gov/pdfs/2003/pdf-index.htm.
(27) Curves - Horizontal Curves. In *FLDOT Plans Preparation Manual, Volume I - English ch. 2: Design Geometrics and Criteria* Florida DOT, 2007, sect. 2.8.1, pp.2-39-2-44. http://www.dot.state.fl.us/rddesign/PPMManual/2007/Volume1/zChap02.pdf.
(28) Horizontal Alignment - Tapers. In *Florida Intersection Design Guide ch. 3: Geometric Design* Florida DOT, 2007, sect. 3.7.2, pp.3-10-3-12. http://www.dot.state.fl.us/rddesign/FIDG-Manual/FIDG2007.pdf.

on superelevated pavements during snow and ice conditions. Use the following guidelines[33]:

- Make sure that the stop sign is perceptible to the driver for a sufficient distance from the intersection to allow deceleration before reaching the curve (see FDM 11-10-5.1.1.4, "Sight Distance on a Stop Sign Controlled Approach").
- Assume a design speed for the horizontal curve of 20 mph less than the sideroad design speed, but not less than 30 mph if the sideroad design speed is less than or equal to 50 mph.
- Limit the superelevation rate on the approach curve to an intersection to 5% or less. The objective is to use as flat an alignment as practical with lower superelevation. The preferred design is to maintain a normal crown section through the curve
- Provide a tangent section prior to the intersection so that the superelevation runoff occurs outside of the intersection radius returns.

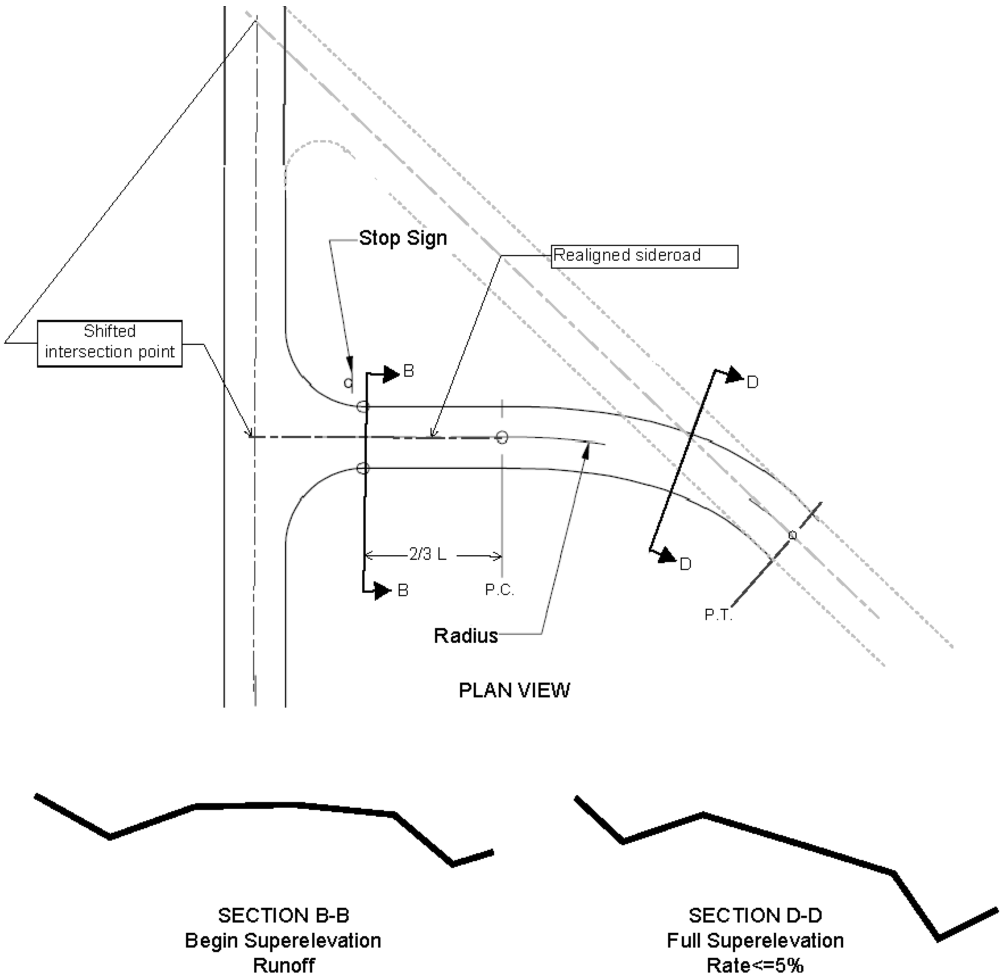

Figure 5.6 Horizontal Curve on a Stop Sign Controlled Approach

5.3 Superelevation

To maintain the desired design speed, highway and ramp curves are generally superelevated.

Superelevation may be defined as the rotation of the roadway cross section to overcome part of the centrifugal force that acts on a vehicle traveling around a curve.

5.3.1 AASHTO revisions

There were several significant changes made to the superelevation guidance in the 2001 AASHTO GDHS and

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[33] Adapted from Illinois DOT Bureau of Design and Environment Manual ch. 36 (*29) ILDOT Bureau of Design and Environment Manual ch. 36*: *Intersections*. Illinois DOT, 2002. http://www.dot.state.il.us/desenv/BDE%20Manual/BDE/pdf/chap36.pdf.

2004 AASHTO GDHS[34]. Some of these changes have affected maximum and minimum curvature "R", runoff length "L", and transition length "T" for a given superelevation rate. These changes include:

- The minimum runoff length is determined solely by applying the maximum relative gradient. It was previously determined by the greater of the distance provided by the maximum relative gradient or the distance provided by 2sec at the design speed.
- The maximum relative gradient is defined as being "*...between the longitudinal grades of the* axis of rotation *and the edge of pavement…*". It was previously defined as being "*...between the edge of two-lane traveled way and the* centerline…"
- The adjustment factor for determining the superelevation runoff on roads other than 2-lanes rotated about centerline is defined based on the number of lanes rotated. It was previously determined based on the width of roadway without regard to the location of the rotation point.
- The following guidance on determining runoff for a Case II median (i.e., a median that is held in a horizontal plane while the two traveled ways are rotated separately around the median edges - see pages 197-198, GDHS 2004 with a width >= 40-feet has been eliminated: *"...the two-lane values should be used for the one-way roadways because the extreme pavement edges are at least 80 ft apart and independent of each other. Values for the one way roadways of six-lane divided highways when separated by a wide median should be 1.2 times the two-lane values..."*
- There is only 1 set of side-friction factors instead of 3.
- The separate equation ($L=47.2*fV_D / C$) used to compute minimum runoff length for low speed urban streets has been eliminated.
- Superelevation tables are now formatted with the superelevation rates in the left hand column and the minimum radii for a given rate and design speed in the body of the table.

5.3.2 Superelevation Rate

This is the rate of rise in cross section of the finished surface of the traveled way of a roadway measured from the lowest or inside edge to the highest or outside edge. AASHTO has established several alternative methods of distributing superelevation and side friction for curve radii that are larger than minimum. Superelevation tables, Exhibit 5.1, provide maximum superelevation rates of 4 and 6 percent. These superelevation rates are based on Method 5 (pages 140-142, GDHS 2004) for distributing superelevation and side friction in the design of all rural highways and high-speed urban streets. See Exhibit 3-27 on pages 169-170, GDHS 2004 for 8-percent superelevation tables. Superelevation rates over 8 percent are not recommended for open highways in areas with ice and snow. Do not use a superelevation rate of less than 2 percent, i.e., reverse crown (RC).

The definitions of the various highway types are as follows:

Rural highway - A highway with a rural cross section and having a posted speed of 50 mph or higher.

High-speed urban highway - Generally, a highway with curb & gutter and having a posted speed of 50 mph or higher.

Transition highway - Generally, a highway with a posted speed of 45 mph that is in a developing area between a rural highway (or high-speed urban highway) and a low-speed urban street.

Low-speed urban street - Generally, a street with curb & gutter and having a posted speed of 40 mph or lower.

Table 5.6, below, provides the WisDOT policy on the use of the various superelevation rates. The use of 8% superelevation in situations other than those shown in Table 5.6 shall be explained in the Design Study Report.

[34] (*3) A Policy on Geometric Design of Highways and Streets 2001*, (2nd printing) 4th edition. AASHTO, 2001. www.transportation.org., ch. 3, pp.131-235, "Elements of Design / Horizontal Alignment"
(*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004., ch. 3, pp.131-231, "Elements of Design / Horizontal Alignment"

Table 5.6 WisDOT Policy on the Use of Superelevation Rate

e_{max}	Areas of Application
8%	For 3R work on roadways where this rate already exists (unless there is a history of crashes related to the superelevation rate).
6%	For new and reconstructed rural roadways and some high speed urban highways [A].
4%	For transition highways and some high speed urban highways [A]
See below	Low-speed urban streets

A. The superelevation rate on high-speed urban roadways is preferably designed using a maximum superelevation rate of 4 percent. Consider adverse effects caused by factors such as, cross street profile, site conditions that include driveways, sidewalks, or other intersections. Six percent superelevation may be used if minimal adverse effects are caused as a result and there are no traffic stops or signals present or anticipated in the future.

For low-speed urban streets select an appropriate superelevation rate from Attachment 5.12 considering design speed, cross slope rate and radius. Superelevation rates for low-speed urban streets should not exceed 4 percent. At lower non-uniform running speeds, which are typical in urban areas, drivers are more tolerant of discomfort, thus permitting employment of an increased amount of side friction (Method 2, pages 140-142, GDHS, 2004) for use in design of horizontal curves. Maximum side friction (f) is shown in Table 5.7 below. Also see pages 148-152GDHS 2004[35].

Table 5.7 Maximum Side Friction (f) Factors (per Exhibits 3-12 and 3-15, GDHS 2004)

Design Speed (mph)	Max. (f)
10	0.38
15	0.32
20	0.27
25	0.23
30	0.20
35	0.18
40	0.16
45	0.15
50	0.14
55	0.13
60	0.12
65	0.11
70	0.10

If the superelevated section is at a signalized intersection the designer must consider the cross street speed, approach grades, and the hump that is created by the superelevation. It may be desirable to flatten the grade. If the superelevated curve is not at an intersecting street/roadway the 4 percent superelevation table, Exhibit 5.1, will provide a higher superelevation rate than superelevation rate using maximum side friction. The following example is provided to show two different ways to determine the superelevation rate on a low-speed urban street:

[35] (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.

Given: V_D = 40 mph

R = 700 ft.

f_{max} = 0.16 (from Table 7)

First solution is obtained from the superelevation tables, e_{max} = 4% (Attachment 9)

R = 700'; e = 0.039 (3.9%).

Second solution is obtained from the nomograph (Attachment 10).

Look on the X-axis, and find 700 feet. Intersect the 700-ft radius line with the 40 mph design curve and read the superelevation rate on the y-axis as –0.75%.

Confirm the nomograph solution by using the **simplified curve formula:**

$e = (V_D^2/15R) - f_{max}$ (English version); or $e = (40^2/15*700) - 0.16$ =

0.152 - 0.16 = -0.0076 (= -0.76%) [checks]

The Attachment 10 nomograph indicates that RC superelevation is to be used when the rate obtained from the nomograph is between –2% and +2%. Therefore, use 2.0% (RC) superelevation.

Where:

V_D = design speed

R = radius

e = superelevation rate

f_{max}= maximum side friction.

In the example, the nomograph indicates a superelevation rate of -0.75%. This is greater than the normal crown slope of -2.0% on the outside of a curve. Therefore, superelevation is needed in this example. However, as noted above, 2.0% (RC) is the minimum superelevation rate to use. Therefore, use 2.0% instead of 0.75%. When the standard table is used for e_{max} = 4%, the table indicates that a 3.9% superelevation is needed. The designer is to use judgment on the use of a superelevation rate (i.e. between 2.0% (RC) and 3.9%) for this specific example situation.

Notice that the results from the nomograph in Attachment 5.12, which is based on the 2004 GDHS[36] (Exhibit 3-17 on p.152), differ considerably from the nomograph in the 2001 GDHS[37] (Exhibit 3-40 on p. 196): -0.75% vs. -2.56%. Low-speed urban streets with existing superelevation that meets the requirements of the 2001 GDHS may retain that superelevation if it is impractical to upgrade to the superelevation obtained from the nomograph in Attachment 5.12, unless there is an unacceptable history of curve related crashes. Document this in the Design Study Report.

Very flat horizontal curves on rural or high-speed urban highways require no superelevation. Traffic entering a curve to the right has some superelevation in the normal crown slope. Traffic entering a curve to the left has an adverse or negative superelevation. Lack of adequate superelevation, where needed, can result in undesirable conditions including: loss of safety factor between the side friction available versus side friction used; driver failure to maintain appropriate lateral position within the lane and increased occupant discomfort. The minimum curve radii for "Normal Crown" which can be designed without superelevation for open road conditions are shown in Exhibit 5.1 for various design speeds.

5.3.3 Superelevation Transition

Superelevation transition is the length required to rotate the cross slope of a highway from a normal crowned slope to a fully superelevated cross slope. See the illustration on Attachment 5.10 and Attachment 5.11. This transition includes a "tangent runout" length needed to remove or add adverse crown. The rotation of the plane of a highway to achieve a superelevated roadway through a horizontal curve begins on the tangent approaches to the curve. WisDOT practice is to place the tangent runout and approximately two-thirds of the length of runoff on the tangent approach and one-third of the length of runoff on the curve. For undivided highways the axis of

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36 (*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.
37 (*3) A Policy on Geometric Design of Highways and Streets 2001*, (2nd printing) 4th edition. AASHTO, 2001. www.transportation.org.

rotation is the centerline of the pavement (see Attachment 5.10). On divided highways, the axis of rotation is normally the median edge of the pavement (see Attachment 5.11). Provide Vertical curves of sufficient length to ensure smooth pavement edges and centerline profile within the superelevation transition (The 2004 GDHS suggests a minimum length in feet equal to the design speed in mph.). These curves may be either computed or determined graphically.

When using the superelevation rate from the tables in Exhibit 5.1, use the corresponding values for “L” and “T” from these tables to design the curve. A small increase in runoff may be appropriate on high-type facilities (freeway, expressway, or other divided highway) in order to facilitate drainage or to smooth out the traveled way edge profile. Any superelevation transition locations that are computed using something other than the superelevation tables and runoff tables provided, must be hand entered into the superelevation spreadsheet for consideration by CAiCE. For the above example the designer was assumed to choose a superelevation rate of 2 percent. Compute the theoretical point of normal crown and the theoretical point of full superelevation.

Given:

PC = Station 870+00.00

L = $(wn_1)e_d(b_w) / \Delta$, where

w = lane width (feet) = 12-feet (use for consistency and practicality even if lane width used does not equal 12-feet);
n_1 = number of lanes rotated = 1;
e_d = design superelevation rate (%) = 2.0%;
b_w = adjustment factor for number of lanes rotated (see Table on page 9 of Exhibit 5.1) = 1.0;
Δ = maximum relative gradient (%) (see Exh 3-30 in 2004 AASHTO GDHS) = 0.58% for 40 mph
therefore, L=12*1*2.0%*1.0 / .0.58% = 41.4 ft (round to 41 ft)

X = e_{NC}L / e_d = 41 * 0.02/0.02 = 41 ft

Theoretical point of normal crown (see Figure 7 and 8):

PC - 2/3L - X = 870+00.00 - 27.33 - 41 =

Station 869+31.67

Theoretical point of full superelevation (see Figure 7 and 8):

PC + 1/3L = 870+00.00 + 13.67 =

Station 870+13.67

Where:

PC = Point of Curvature

L = Length of Runoff

X = Length of Tangent Runout

e_{NC} = Normal Crown of 2%

e_d = superelevation rate

The EDIT SUPERELEVATION for alignment command in CAiCE can be used to create transition lines at these computed locations.

Avoid superelevation transition on a bridge because it complicates bridge design and construction. When superelevation must be partially developed on a bridge, there should be a clear understanding between the bridge designer and the roadway designer as to the method used for transition development and the resulting grades.

Auxiliary lane pavement (or right turn lane) on the high side of superelevated curves shall be maintained at the same slope as the adjacent traffic lane until the superelevation reaches 4 percent. When superelevation on the traffic lane pavement is greater than 4 percent the auxiliary lane slope will remain constant at 4 percent. In some isolated situations where the increasing elevation of the intersecting side road approaches the main line road, it may be desirable to flatten the superelevated auxiliary lane to form a gradual transition between the superelevated section and the side road. Do not exceed a rollover rate greater than 5 percent between adjacent travel lanes or auxiliary lanes.

Do not exceed a rollover rate greater than 8 percent between shoulder and travel lanes or auxiliary lanes.

On a superelevated divided highway where a “narrow” median is present, it may be desirable to rollover the high side shoulder and bring up the median shoulder to reduce the elevation difference between the divided highways. This special situation may be desirable in an urban condition when the highways are divided by a barrier wall.

5.4 Vertical Alignment

The highway vertical alignment consists of tangents or grades and vertical curves. Vertical curves are based on sight distance considerations. Headlight sight distance is the primary factor used to determine the length of sag vertical curves (see Attachment 5.6 and Attachment 5.7).

Although grade changes without a vertical curve are discouraged, there may be situations where it is necessary. This must be explained and justified in the DSR. Table 5.8 shows the maximum change in grade without a vertical curve. Some rounding of the deflection point is anticipated during construction.

Table 5.8 Maximum Change in Grade Without a Vertical Curve

Design Speed mph	20	30	40	45	50	60	65	70
Maximum Change in Grade in Percent	1.20	1.00	0.80	0.70	0.60	0.40	0.30	0.20

5.4.1 Grades

Maximum grades (see Attachment 5.3 of this procedure and FDM 11-15 Attachment 1.4) vary with terrain, design speed and functional classification.

The minimum grade on roadways with rural cross sections is 0.0 percent, i.e., flat, except in areas of superelevation transition and other areas with pavement rotation. Do not use flat grades in areas of superelevation transition and other areas with pavement rotation because the combination of a flat longitudinal grade with a flat cross-slope results in pavement surface drainage problems. Provide a minimum grade in these areas based on AASHTO guidance for “Minimum Transition Grades”[38]. This applies to both rural and urban roadways.

If grades of less than 0.5 percent are used, then side ditches should be specially designed to provide sufficient longitudinal gradient for drainage. On divided highways, grade lines of opposing roadways should be treated independently except where topographic or other conditions require them to be identical. The minimum gradient on structures is 0.5 percent to ensure positive drainage.

Compatibility of curb and gutter grades with the existing development is essential in reducing damage to abutting property and the amount of right-of-way to be acquired. To ensure drainage the minimum gradient of curb and gutter is desirably 0.50 percent but at least 0.30 percent. Special attention may be required to assure proper drainage of curbed pavements at the apex of crest vertical curves where a level point occurs. Drainage should be adequate for vertical curves having "k" values of 167 or less (see Attachment 5.4, Attachment 5.5 and pages 270, & 274, GDHS 2004).

An exception to standards is required for grades that are either greater than maximum or less than minimum.

5.4.1.1 Climbing Lanes

See FDM 11-15-10 for guidance on climbing lanes.

5.4.2 Vertical Curves

Design vertical curves to provide adequate sight distance, safety, comfortable driving, good drainage, and pleasing appearance. They are normally symmetrical parabolas. A notable exception would be the use of an asymmetrical parabolic curve to provide better drainage of a structure located on a crest vertical curve.

Vertical curves are generally identified by their “K” values. K is the rate of curvature and is defined as the length of the vertical curve (L) divided by the algebraic difference in grade (A); i.e. the horizontal distance in feet required for a 1 percent change in gradient. K is affected by sight distance, comfort, drainage, and aesthetic quality. Sight distances and vertical curve k-values are shown in Attachment 5.4 through Attachment 5.7 for each of the sight distance categories discussed earlier in FDM 11-10-5.1.1.1 “Application of Stopping Sight Distance (SSD) and Decision Sight Distance (DSD)”. Crest vertical curve values are shown in Attachment 5.4

[38] See(*1) A Policy on Geometric Design of Highways and Streets 2004*, 5th edition. AASHTO, 2004.

and Attachment 5.5. Sag vertical curve values are shown in Attachment 5.6 and Attachment 5.7.

Compute sight distance (S) on a vertical curve by re-arranging the equations on Attachment 5.4 and Attachment 5.6 to solve for (S). Note that as a vehicle traverses a vertical curve, the distance to the end of the vertical curve (i.e., the L-dimension in the equation) gets shorter. The A-dimension also decreases because A=L/K and K is constant. The sight distance on the vertical curve begins to increase when the vehicle reaches a point where S>L. Consider this when determining if the requirements for a sight distance category are met.

A design exception for vertical alignment is required for a crest vertical curve if:

- It does not provide minimum sight distance for the sight distance category (a design exception may also be required for stopping sight distance)

A design exception for vertical alignment is required for a sag vertical curve if:

- It does not provide the minimum headlight sight distance and adequate street lighting is not provided (a design exception may also be required for stopping sight distance), or
- It does not meet the comfort criteria for sag vertical curves

On vertical curves with K > 167, there will be a section of roadway at least 100 feet in length near the crest or sag with a grade of less than 0.30%. This condition may create drainage problems, especially on curbed highways. It is not intended that K of 167 feet per percent grade be considered a design maximum, but merely a value beyond which drainage should be more carefully designed.

5.4.3 Vertical Clearance

See FDM 11-35, Attachment 1.8 and Exhibit 5.1 for vertical clearance requirements for different combinations of overpass and underpass facilities.

Vertical clearances for overhead utility facilities shall comply with all applicable state and national electrical codes. See WisDOT Highway Maintenance Manual chapter 96 (http://www.dot.wisconsin.gov/business/rules/property-96.htm) and WisDOT LRFD Bridge Manual chapter 3 for vertical clearance for overhead utilities. (http://on.dot.wi.gov/dtid_bos/extranet/structures/LRFD/LRFDManualIndex.htm)

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LIST OF ATTACHMENTS

Attachment 5.1	Sight Distance Values
Attachment 5.2	Sight Distance Categories and Applications
Attachment 5.3	Maximum Grades by Functional Classification
Attachment 5.4	Sight Distance for Crest Vertical Curves

LIST OF EXHIBITS